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SEISMIC BRIDGE RESPONSE TO DIFFERENTIAL GROUND MOTION AND COMPARISON WITH ALGERIAN SEISMIC DESIGN RULE.

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Abstract. *Seismic response of extended structures, such as bridges, must take into account spatial variability of earthquake ground motion (SVGM). Models that describe SVGM have been developed during last decades and structural analyses have been performed on numerous structures. Based on these studies, simplified approaches have been developed. The new Algerian bridge seismic regulation code proposes a simplified approach that takes into account SVGM. This paper aims at performing preliminarily studies on the accuracy of this method. This is performed through comparison with more refined approaches. Bridges, with different overall lengths and seating on different types of site conditions, are considered. The results show that this simplified method overestimates the response of the analyzed bridges* **.**

1 INTRODUCTION

Seismic analysis of extended structures, such as bridges, must take into account spatial variability of earthquake ground motion (SVGM) which can induce significant additional forces. In fact, it has been recognized that space-time variability of the seismic of ground motion is the results of three distinct effects [2]: (1) loss of coherence of the seismic movement due to multiple refractions and reflexions of the seismic waves along their paths, named incoherence effect, (2) difference in arrival times of the seismic waves at the various recording stations due to the variation of their apparent propagation velocity, named wave passage effect, (3) space variation of the geotechnical properties, named site effect.

During last decades, models that describe SVGM have been developed based on either empirical or analytical approach (as e.g., [3, 4]) and it is now widely accepted that the coherency function describes the SVGM. Using these models, structural analyses have been performed on numerous structures and show the importance of taking into account SVGM as, e.g., among many others, in references: [5, 6, 7, 8, 9, 10]. Based on these studies, simplified approaches have been introduced in regulation codes to take into account SVGM. The Algerian bridge seismic regulation code (RPOA) [1] has been recently lunched.

This paper aims at performing preliminarily studies on the accuracy of the simplified approach proposed by the RPOA. For the purpose of the study, seismic motion is simulated using time domain segmentation and the well-established method of Deodatis [11]; SVGM is described by the empirical model of Harichandran and Vanmarcke [3]. The simulated time histories are used as input excitations at bridge supports. The time-history analysis results are compared to those obtained by the simplified method of RPOA for different bridges. Results are presented in terms of internal forces.

2 SIMULATION OF SPATIALLY VARYING GROUND MOTIONS

In order to study the effect of SVGM on bridges responses, it is necessary to generate acceleration and displacement time-histories at several locations on the ground surface, corresponding to the bridge supports. In this study, the seismic ground motions are simulated as non stationary from predefined time history, using time domain segmentation method [12, 13, 14]. The predefined time histories are divided into nearly stationary segments with different durations. Then, each segment is used as a reference time series and stationary conditional simulations are carried out for each segment, using the simulation technique proposed by Deodatis [11]. The simulated segments are joined together to obtain the entire non stationary and spatially variable acceleration time-histories, after the incorporation of a time shift to account for the wave passage effect.

The generated acceleration time histories are further corrected and integrated in order to obtain the corresponding displacement time-histories. The properties of each set of simulated time-histories are the same in terms of target power spectral density function, peak of displacement and response spectrum compatibility.

3 OVERVIEW OF THE RPOA PROVISIONS REGARDING SVGM

RPOA [1] is the first Algerian code established for the seismic design of bridges. RPOA clearly recognizes that, since the differential ground motion induces significant internal forces, the seismic action cannot be based only on the characterization of uniform motion, and proposes a simplified approach to take into account SVGM effects. According to RPOA, the effects of differential displacements between supports are generally negligible for current structures, except when: (a) the structure crosses an active fault, (b) the soil properties vary along the bridge, (c) the length of the bridge is very important.

According to RPOA, designers must perform, firstly, a dynamic analysis of the structure under uniform seismic excitations, and secondly, a pseudo-static analysis based on pattern of prescribed differentials displacements at the bridge supports. Finally, the dynamic response is combined with the pseudo-static response using the SRSS rule.

On a ground site without significant mechanical discontinuity, the design differential displacement *d*, between two points separated by a distance *X*, is given by [1]:

$$d = \eta A g X;\ \text{for}\quad X < L_M \tag{1}$$

$$d = A g D_M \sqrt{2}\,;\text{for}\quad X \geq L_M \tag{2}$$

With:

$$\eta = \frac{D_M}{L_M}\sqrt{2} \tag{3}$$

Ag: is the design seismic acceleration; g is the acceleration of gravity.

L_M : is the distance beyond which the motions of the two supports can be regarded as independent.

D_M: are absolute displacements; they are given for unit acceleration (1m/s²).

The values of D_M and L_M are given in Table 1 for the four ground types in RPOA, S1 to S4, which are classified on the basis of the shear wave velocity V_s.

Ground Type	S1	S2	S3	S4
$V_s\ (m/s)$	$V_s \geq 800$	$400 \leq V_s \leq 800$	$200 \leq V_s \leq 400$	$V_s \leq 200$
$L_M\ (m)$	600	500	400	300
$D_M\ (m)$	0.03	0.05	0.07	0.09

Table 1. Values of L_M and D_M [1]

RPOA do consider two special situations. If two support points are located on both sides of a significant topographic discontinuity (valley), and in absence of a better approach, the value of *d* is raised by 50%. In case they are located on both sides of a mechanical discontinuity (fault), design differential displacement *d* is given by:

$$d = Ag\sqrt{D_{M,1}^2 + D_{M,2}^2} \tag{4}$$

Where:

$D_{M,1}$ and $D_{M,2}$ are the absolute displacements at points 1 and 2.

4 EVALUATION OF THE SIMPLIFIED APPROACH OF RPOA

4.1 Bridge model

In order to quantitatively assess the simplified method of RPOA, two bridges are selected (Bridge A and Bridge B). They have different overall lengths (i.e.400m and 600m, respectively); they have the same configuration (Figure 1) which is taken from design example No. 1 from the Federal Highway Administration seismic design examples [15]. The span length is constant and equal 50 m; the respective numbers of spans are 8 and 12.

The finite element models use six equal-length 3-D elastic beam elements per spans and four beam elements per pier. The superstructure and the columns are connected by rigid elements. The shear stiffness of the bearings is assumed to provide no restraint in the longitudinal direction. In the vertical direction, the bearings are considered fully restrained due to the gravity forces of the superstructure. The rigid element at each end of the bridge is restrained in the transverse direction by springs, which represent the effect of the girder stops at both ends of the bridge. The stiffness of each bent foundation is modeled by six springs at the lower end of the footing elements, which were determined using an elastic half-space approach [15]. Finally, 5% Rayleigh damping is utilized.

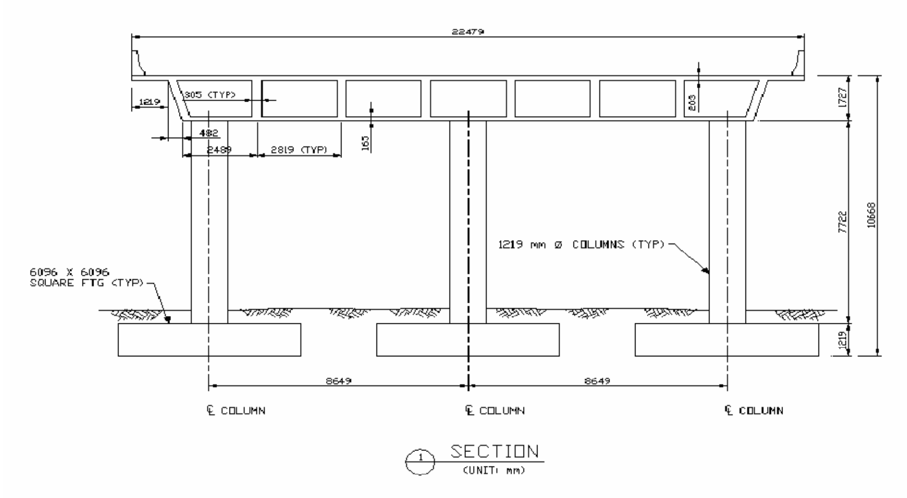


Figure 1. Girder cross-section.

4.2 Support motions

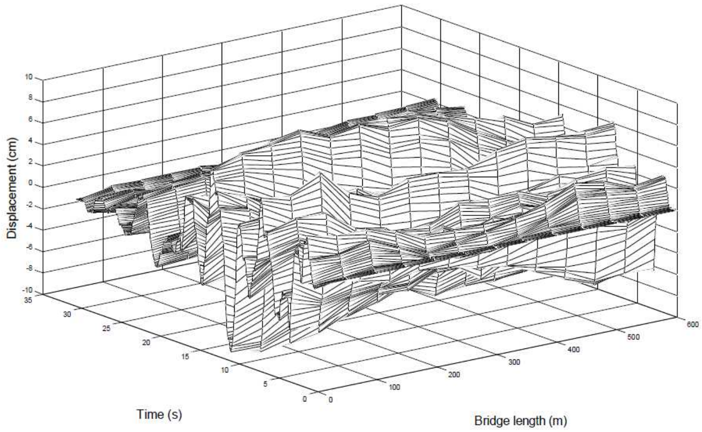
In this study, uniform soil conditions are assumed and only the longitudinal component of the excitation is considered. The accelerograms used for the conditional simulation of support motions are compatible with RPOA's response spectrum. Two ground types S1 (firm soil) and S3 (soft soil), 5% damping and 0.4g peak ground acceleration are selected to describe the ground motion at piers. The Harichandran and Vanmarcke model [3] is chosen to model the coherence loss between pair of bridge supports:

$$\left|\gamma_{jk}(\omega, d_{jk})\right| = A.exp\left(-\frac{2(1-A+\alpha A)|d_{jk}|}{\alpha\theta(\omega)}\right) + (1-A).exp\left(-\frac{2(1-A+\alpha A)|d_{jk}|}{\theta(\omega)}\right) \tag{5}$$

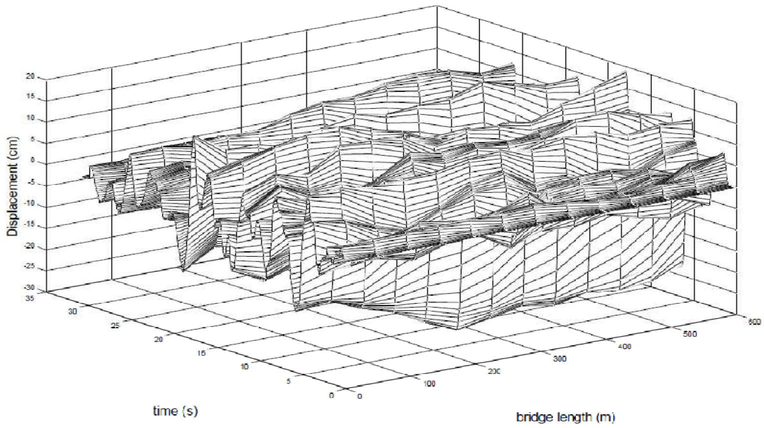
$$\theta(\omega) = k\left[1 + \left(\frac{\omega}{2\pi\omega_0}\right)^b\right]^{-\frac{1}{2}} \tag{6}$$

The following parameters of the model are used: $A = 0.736$, $\alpha = 0.147$, $k = 5210\ m$, $\omega_0 = 6.85\ rad/s$ and b=2.78, which correspond to data recorded during Event 20 at the SMART-1 array, Lotung, Taiwan. Since the span length is the same for all bridges, it was decided to simulate stationary SVGM every 50 m. In this study, an apparent propagation velocity $v = 750\ m/s$ was used.

For Monte Carlo simulation needs, the procedure of simulation is repeated 10 times. Fig. 2 gives one set of non stationary SVGM displacements corresponding to ground types S1 and S3 which were simulated for the longest bridge (600 m, i.e., 13 support points).



(a)



(b)

Figure 2. One sample of non stationary SVGM displacements
for ground types (a) S1 and (b) S3.

4.3 Analysis Results

In order to evaluate RPOA's simplified approach, the bridges presented in section 4.1 are analyzed three times, using the following three cases of analysis:

URSA: Conventional response spectrum analysis, i.e., which assumes uniform ground motion, using RPOA's response spectrum.

VTHA: Time-history analysis using the asynchronous displacements simulated in section 4.2.

VRPA: RPOA's simplified analysis for SVGM (see section 3) using RPOA's response spectrum. The prescribed differential displacements are calculated using Eqs. (1) to (3) and are presented in Table 2.

Tables 3 and 4 present comparisons of the bending moment demand envelopes at the extreme column of each bent of Bridge A. It should be noted that VTHA results are the mean values obtained from 10 time-history analyses. These tables show that VTHA results remain equal or lower than those obtained using conventional response spectrum analysis (URSA). Consequently, in this case, the effect of SVGM is negligible (i.e. lower than 5%), and to a certain degree, it is even beneficial (i.e. a reduction in the resulting bending moments is observed). The latter is an observation in agreement with the findings of previous studies [16], for the symmetric bridge configuration and uniform soil condition. However other studies as, e.g.,[17, 6, 8], observed that this findings cannot be generalized, and concluded that, depending on the characteristics of the SVGM, the bridge configuration and its boundary conditions, spatially variable ground motions can induce a higher or lower response in the structure than the response resulting from uniform ground motions.

Site		***S1***	***S3***
η		$0.7x10^{-4}$	$2.5\ x10^{-4}$
$\boldsymbol{L_M}$		*600*	*400*
d **(m)**	Abut A X=0	0	0
	Pier 1 X=50m	0.013	0.049
	Pier 2 X=100m	0.027	0.098
	Pier 3 X=150m	0.041	0.147
	Pier 4 X=200m	0.055	0.196
	Pier 5 X=250m	0.068	0.245
	Pier 6 X=300m	0.082	0.294
	Pier 7 X=350m	0.096	0.343
	Pier8 X=400m	0.11	0.388
	Pier9 X=450m	0.123	0.388
	Pier10 X=500m	0.137	0.388
	Pier11 X=550m	0.151	0.388
	Abut BX=600m	0.165	0.388

Table 2. Differential displacements for Bridge A and Bridge B, according to RPOA.

In addition, tables 3 and 4 suggest that VRPA and VTHA give comparable results: they reduce the seismic demand in the central piers and increase it in piers close to abutments. However it is found that VRPA amplify the results and the differences are more pronounced in the lateral piers. VRPA can amplify the results by 20% for the firm soil and 50% for the soft one.

Table 5 presents ratios between pier top displacements given by VRPA and VTHA. It is found that VRPA displacement amplification is as high as 1.44 and 1.82, for ground types S1 and S3, respectively

The results for Bridge B are presented in Tables 6-7. Once again, it is observed that VRPA overestimates the seismic demand. It reaches 37% for the firm ground type and 55% for the soft one, which is higher than those observed for Bridge A.

Pier maximum bending moment (MN.m)	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
URSA	13.07	13.95	13.62	13.70	13.62	13.95	13.07
VTHA	13.75	12.74	12.70	09.87	09.75	10.13	11.84
VRPA	14.65	14.53	13.69	13.75	14.07	15.31	15.7
Ratio							
VTHA/URSA	1.05	0.91	0.93	0.72	0.71	0.72	0.90
VRPA/URSA	1.12	1.04	1.00	1.00	1.03	1.09	1.20

Table 3. Bending Moment demand envelopes: Bridge A- S1.

Piers maximum bending moment (MN.m)	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
URSA	24.75	26.22	25.64	25.85	25.64	26.22	24.75
VTHA	24.95	22.97	20.90	19.10	17.81	21.13	22.50
VRPA	32.38	34.02	26.02	26.11	28.25	33.39	37.53
Ratio							
VTHA/URSA	1.00	0.87	0.81	0.74	0.69	0.80	0.9
VRPA/URSA	1.30	1.48	1.01	1.01	1.10	1.27	1.51

Table 4. Bending Moment demand envelopes: Bridge A- S3.

Piers Top displacement Ratio	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6	Pier 7
VRPA/VTHA (S1)	1.34	1.36	1.37	1.39	1.41	1.43	1.44
VRPA/VTHA (S3)	1.67	1.69	1.71	1.73	1.76	1.79	1.82

Table 5. Comparison of Pier Top Displacement: BridgeA-S1 and S3.

Pier maximum bending Moment (MN.m)	Pier1	Pier2	Pier3	Pier4	Pier5	Pier6	Pier7	Pier8	Pier9	Pier10	Pier11
URSA	13.12	14.01	13.65	13.70	13.69	13.66	13.69	13.70	13.65	14.01	13.12
VTHA	12.82	12.2	12.39	9.22	8.49	7.78	8.36	9.05	8.78	10.16	11.66
VRPA	16.59	16.03	14.67	14.05	13.74	13.69	14.04	14.7	15.57	17.33	18.01
Ratio											
VTHA/URSA	0.97	0.87	0.90	0.67	0.62	0.57	0.61	0.66	0.64	0.72	0.88
VRPA/URSA	1.26	1.14	1.07	1.02	1.00	1.00	1.02	1.07	1.14	1.23	1.37

Table 6. Absolute Moment demand envelopes of the bridge pier: Bridge B-S1.

Pier maximum bending Moment (MN.m)	Pier1	Pier2	Pier3	Pier4	Pier5	Pier6	Pier7	Pier8	Pier9	Pier10	Pier11
URSA	24.79	26.28	25.68	25.81	25.76	25.75	25.76	25.81	25.68	26.28	24.79
VTHA	22.71	22.54	21.29	18.52	14.98	15.42	14.4	16.09	18.27	20.94	23.41
VRPA	38.55	34.16	29.23	26.65	25.77	26.94	29.99	34.10	33.42	33.73	31.81
Ratio											
VTHA/URSA	0.91	0.85	0.82	0.71	0.58	0.59	0.56	0.62	0.71	0.79	0.94
VRPA/URSA	1.55	1.29	1.14	1.03	1.00	1.04	1.16	1.32	1.30	1.28	1.28

Table 7. Absolute Moment demand envelopes of the bridge pier: Bridge B- S3.

5 CONCLUSIONS

RPOA gives a simplified approach to introduce the effect of the SVGM in the design of bridges. In this paper, the accuracy of this method is evaluated through comparison with a more refined approach. Two bridges, having different lengths and seating on different types of ground conditions, are considered. For each bridge/site case, three types of linear analyses are conducted. Based on this study, the following conclusions can be done:

The spatial variability of earthquake ground motion can significantly change the structural response. SVGM increases seismic demand in some cases and reduces it in others. The present study clearly demonstrates that the simplified method does not give satisfactory results and overestimates the seismic demand, especially for laterals piers. In order to reduce those results it is suggested to decrease the differential displacement given by RPOA code.

It should be noted that the present analysis correspond only to one model of bridge with different overall lengths. However, additional research needs to be conducted in this area for enrichment of the presents study.

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